Short Communication

Design of Relief Wells at Karanjwan Dam

by

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Introduction

Karanjwan dam, built across river Kadva, is one of the storage dams which constitute the Upper Godavari Project. Karanjwan dam consists of a masonary spillway and non-overflow portion in the gorge and earthen embankments on the flanks. The earth embankment 900 m long and of average depth of 17m on the left bank is called Karanjwan main earth dam. From the drill hole log it is observed that the rock level is quite deep, maximum 28m below ground level. The soil deposited is brought by the river and a buried valley is formed.

The earth work was completed by March, 1974. The reservoir was filled upto about 7 m height above average ground level because of paucity of rainfall in that year. The full storage height is 16 m. Air bubbles appeared at the end of rainy season (24th Oct., 1974) at Ch. 1040 in the collecting drain at the downstream of the earth dam. Bubbles were noted to have spread all over the downstream area by 25th and 26th October. On 27th November 1974 a crator formation was noted in the cross drain nearby Ch. 1040. It was observed that the seepage water had residual head upto one helf metre and more. A stand pipe at Ch. 1057 m downstream 75 m was overflowing since 16.9.1974 with a discharge of about 4 litres/min. (Bhave and Moghe, 1978). Relief wells were provided at the toe of the dam as a remedial measure.

Permeability Observations

A number of field permeability tests were carried out in bore holes at this dam site and on the bases of the test data it was decided to provide a partial cut off trench below the earth dam. The L-section of the dam showing soil stratigraphy along with the values of coefficient of field permeability is illustrated in Figure 1. The magnitude of coefficient of permeability rangss from 2×10^{-6} cm/sec to 50×10^{-6} cm/sec, the highest being 149×10^{-6} cm/sec. The values of coefficient of permeability are quite low. It is noted that the coefficient of permeability increases with depth and it is less than 5×10^{-6} cm/sec for a depth ranging from 53 per cent at Ch. 450 to 90 per cent at Ch. 1175 indicating that the top 50 per cent to 90 per cent depth of the foundation is impervious.

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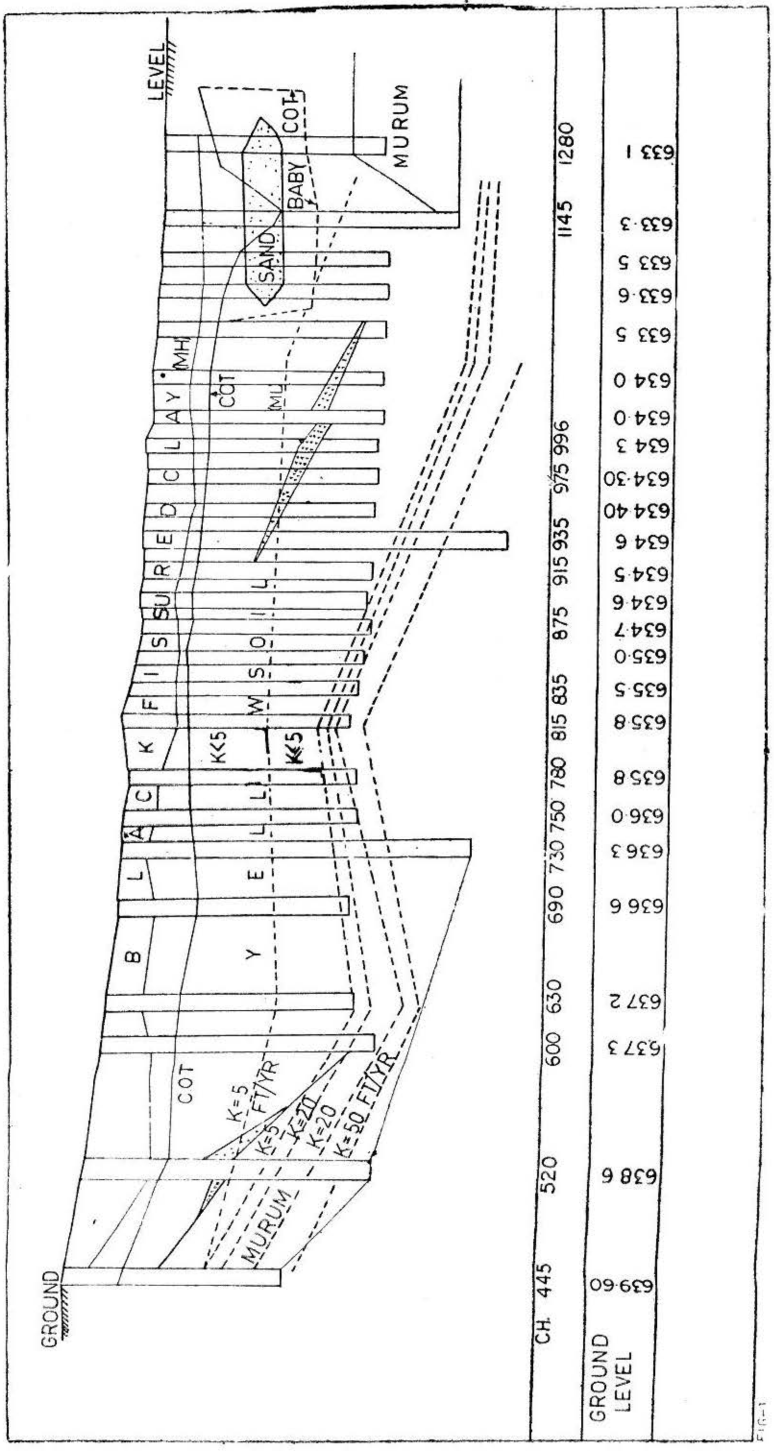

FIGURE 1 Profile of the area studied

At ch. 1170, sand layer was met at a depth of 5 m during excavation for cut off trench. The cut off trench was taken below this layer between Ch. 1085 to 1200 (Figure 1).

Some sand pockets which were not located during drilling of bore for investigation purpose were logged at the time of drilling bore holes for relief wells. In the relief well at Ch. 445 a sand layer was observed 6 m below ground level. The murum layer met with at 6 m depth located below sand layer was also previous. The murum layer at 9.35 m logged in bore hole at Ch. 520 was previous. A sand pocket from Ch. 935 to Ch. 1071 with depth varying from 7.0 m to 14.0 m as one moves from Ch. 935 to 1071 was located at this time. The maximum thickess of the sand layer is approximately one meter at Ch. 1025 m.

The design of wells yielded the distance between wells depending on different depths of penetration of wells as shown in Table 1.

TABLE 1

Details Regarding Wells

Sr. No.	Distance between wells (m)	No. of wells	Depth of penetratation of wells (m)
1	50	15	28
2	20	37	7.784
3	28	27	12.208
4	38	20	14.548

In view of the face that the depth of the previous layers vary from 6 m to 14 m, it is recommended to provide relief wells of 12 meters depth or 16 meters depth with corresponding numbers of wells 27 or 20. The actual numbers of wells provided are 25, out of which six wells are fully penetrating and nineteen wells are partially penetrating upto 15 m depth as shown in Figure 1.

The Critical Head

In order to recognise the possibility of the formation of boils which may consequently lead to the piping failure of the earth dam, the magnitude of the critical head which may be responsible for boiling action is required to be calculated.

It is to be noted that the coefficient of permeability has no influence on the value of critical head. Thus for a given geometery of the earth dam, depth of penetration of cut off trench and the depth of foundation soil above rock level, critical height of water head is same irrespective of permeability of foundation soil. The factor that affects the geometrical shape of the earth dam is hydraulic anisotropy of the foundation soil as the magnitude of excess hydrostatic pressure will change when the shape of the earth dam section along with its foundation change in transformation so as to treat the foundation soil as isotropic.

Table 2 gives the magnitude of critical head with variation in $\sqrt{\frac{KH}{KV}}$ from 1 to 8.

TABLE 2

Magnitudes of critical head with $\sqrt{KH/KV}$

$\sqrt{\frac{KH}{KV}}$	Magnitude of critical head m
1	40.0
3	21.0
6	16.25
8	10.4
10	8.0

It is found that in order to develop boiling action at 8 m hydraulic head the magnitude of $\sqrt{\frac{KH}{KV}}$ should be 10.

The above study indicates that the ratio of coefficient of permeability of foundation soil in horizontal and vertical direction is almost 100 m. The boiling action at the downsteam toe of the dam was started because of high hydraulic anisotropy of the foundation soil and is independent of the magnitude of the coefficient of permeability.

Permeability of Foundation Soil

The magnitude of coefficient of permeability (Figure 1.) as determined by point permeability tests of foundation soil are rather low. It was thought that these values of coefficient of field permeability might be much smaller than the actual ones, as the boiling action took place immediately after the first filling of the reservoir.

With the knowledge of rate of quantity of flow from each relief well, it is possible to determine average horizontal permeability of the foundation soil nearby that particular relief well. Table 3 gives this information. It is noted that the average coefficient of permeability varies from 1.98×10^{-2} cm/sec at Ch. 1071.5 to 5.98×10^{-4} cm/sec at Ch. 650 m. This gives the range of magnitude of coefficient of permeability which is very high.

Highly plastic black fissured clay is present as a top stratum. The effect of smearing, while drilling bore holes, which forms an impervious

TABLE 3

Average Cofficient of Permeablities

Sr. No.	Chainages	K (1975) cm/sec	K (1977) cm/sec	K (1979) cm/sec.
1	815	8.9×10^{-3}	1.8×10^{-5}	3.1×10^{-5}
2	835	2.6×10^{-3}	1.2×10^{-5}	6.4×10^{-5}
3	855	1.0×10^{-2}	7.6×10^{-5}	8.0×10^{-5}
4	875	2.7×10^{-3}	1.2×10^{-5}	7.7×10^{-6}
5	895	4.4×10^{-3}	1.5×10^{-5}	1.2×10^{-5}
6	915	4.3×10^{-3}	1.1×10^{-5}	2.4×10^{-5}
7	935	4.0×10^{-3}	2.7×10^{-5}	1.8×10^{-5}
8	955	1.5×10^{-3}	1.2×10^{-5}	4.3×10^{-5}
9	975	5.6×10^{-4}	6.0×10^{-6}	1.8×10^{-6}
10	995	1.8×10^{-3}	1.2×10^{-5}	5.8×10^{-6}
11	1016	3.3×10^{-3}	2.5×10^{-5}	1.8×10^{-5}
12	1041.5	2.7×10^{-3}	1.4×10^{-5}	1.3×10^{-5}
13	1071.5	1.9×10^{-2}	1.2×10^{-4}	1.3×10^{-5}
14	1095.0	1.3×10^{-4}	1.3×10^{-4}	1.3×10^{-4}
15	1145.0	1.2×10^{-5}	4.0×10^{-6}	1.4×10^{-6}

cake on the surface of the bore hole might be the reason that the magnitudes of point permeabilities obtained were smaller than those determined from the rate of flow from the relief wells. It was observed that the magnitudes coefficient of permeability decreases with time, from the year 1975 to 1979, as shown in Table 3. The coefficient of permeability of foundation soil is 10^{-3} on average as obtained from the quantities of discharge observed in the year 1975 at maximum lake level. The magnitudes of coefficient of permeability at maximum lake level in the year 1975 shows, on average, decrease to 10^{-5} cm/sec. The decrease in K value from 1977 to 1979 is small. The conspicious decrease in the coefficients of permeability from the year 1975 to 1977 may be due to formation of an impervious blanket on the upstream due to the deposition of silt. The decrease in the discharge quantity is not due to decrease in the efficiency of wells as these were surged and cleaned every year. In addition it was found that tail water level is not rising with years 1975 to 1979 as this would have been obviously the case if the relief wells were not functioning efficiently.

Conclusions

The determination of coefficient of field permeability by point permeability method should be done with proper care and must be improved, taking into cosideration the smearing effect of upper clay soil in decreasing the values of field permeability. It is better to conduct large scale field permeability tests like Thiems's test so as to understand the overall permeability of a large area of foundation soil for its full depth.

It is necessary to determine the anisotropy in permeability in horizontal and vertical directions, particularly when the soil is deposited by the river as in the present case. The magnitude of the critical head responsible for development of boiling action is mainly influenced by the ratio of horizontal to vertical permeability.

The coefficient of permeability of the foundation soil decreased considerably after three years of service of the dam, possibly because of the formation of upstream impervious blanket due to silting. The danger of boiling and subsequent piping on the downstream of the dam is successfully avoided by the installation of relief wells.

Acknowledgment

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References

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